Update to a Geologic Hazards and Geotechnical Investigation
Tax Lots 1300 and 1800, Map 7-11-34BA
The Pointe
Lincoln City, Oregon

Prepared for:
Northwest Coastal Investments
[redacted]

Project #Y184132C

April 18, 2023

H.G. Schlicker & Associates, Inc.

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Project #Y184132C April 18, 2023

To: [redacted]

Subject: **Update to a Geologic Hazards and Geotechnical Investigation**
Tax Lots 1300 and 1800, Map 7-11-34BA
The Pointe
Lincoln City, Oregon

Dear Mr. Thompson:

The accompanying report presents the results of our update to a geologic hazards and geotechnical investigation for the above subject site.

After you have reviewed our report, we would be pleased to discuss it and to answer any questions you might have.

This opportunity to be of service is sincerely appreciated. If we can be of any further assistance, please contact us.

H.G. SCHLICKER & ASSOCIATES, INC.

Adam M. Large, MSc, RG, CEG
President/Principal Engineering Geologist

AML:mgb

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Tax Lots 1300 and 1800, Map 7-11-34BA
The Pointe
Lincoln City, Oregon

Dear Mr. Thompson:

1.0 Introduction and General Information

At your request and authorization, representatives of H.G. Schlicker and Associates, Inc. (HGSA) visited the subject site on March 17, 2023, to complete an update to a geologic hazards and geotechnical investigation of Tax Lots 1300 and 1800, Map 07-11-34BA, The Pointe, Lincoln City, Oregon (Figures 1 and 2; Appendix A). It is our understanding that you are planning to construct a new house generally on Tax Lot 1300 and develop Tax Lot 1800 with a swim spa and attached deck.

We previously visited the site in 2018 to complete a subdivision-level investigation, which included subsurface exploration and geotechnical analysis. At that time, Tax Lot 1800 was overgrown with thick brush, which made ground surface observations difficult. In 2019 we were called to the site to make additional observations after brushing and clearing had occurred. It is our understanding that a lot line adjustment of the boundary between Tax Lot 1300 and Tax Lot 1800 occurred in 2021. In January of 2023, we met you at the site to discuss potential hazards related to removing trees on the site and the adjacent property. It was our understanding that you are working with a certified arborist and the adjacent property owners to address hazardous trees. We have provided feedback regarding the trees and slope stability concerns in Section 8.3 below.

This report addresses the engineering geology and geologic hazards at the site with respect to constructing a new house and developing the site with a swim spa. The scope of our work consisted of a site visit, site observations and measurements, slope profiles, a limited review of the geologic literature, interpretation of topographic maps, lidar, and aerial photographs, review of our previous reports for the project, geotechnical analysis(as necessary), and preparation of this report, which provides our findings, conclusions, and recommendations.

2.0 Site Description

The site consists of Tax Lots 1300 and 1800, totaling approximately 1 acre. The site is bounded by developed lots to the north, west, and east and to the south by the beach and the mouth of Siletz Bay. SW Coast Avenue provides access.

2.1 Proposed Development

Based on the information provided to us, you plan to construct a house on Tax Lot 1300 and a swim spa with an attached deck is proposed on the southern portion of Tax Lot 1800. We have provided geotechnical recommendations for design of these structures and improvements in Sections 8.0 through 8.13 below. HGSA should be contacted to review development plans for the site. There will be additional charges for these services.

2.2 History of The Site and Surrounding Areas

An aerial photograph from April 2007 shows a house in the area of Tax Lot 1300. This house was removed, and subsequently, the site underwent grading. S.W. Coast Avenue and S.W. Beach Avenue are now connected.

The subject property does not have an oceanfront protective structure, and lies in an area of high bluffs that generally lack oceanfront protective structures. According to the Oregon Coastal Atlas Ocean Shores Data Viewer (http://www.coastalatlas.net/oceanshores, accessed April 2023), Tax Lot 1800 is not eligible for a beachfront protective structure on the Goal 18 Eligibility Inventory. Tax lot 1300 is not an oceanfront property and is outside the inventory mapping.

2.3 Site Topography, Elevations, and Slopes

The lots are situated along a south-southwest facing bluff on the northern side of the mouth of Siletz Bay. Lidar derived elevations at the site range from approximately 10 to 18 feet along the beach to 108 feet (NAVD 88) in the upper portion northwest portion of the site (Figures 3 and 4). There is an approximately 40 to 70-foot-high bluff along the site's southern portion that slopes down at approximately 40 to 70 degrees. Generally, the site's northern portion slopes to the east from approximately 5 to 25 degrees. However, steep slopes, up to approximately 30 feet high with portions sloping near vertical, occupy the eastern portion of the site.

2.4 Vegetation Cover

The upper portions of the site are moderately vegetated with trees and low brush. Clearing and grading activities have affected most of the ground surface on the northern portions of the site.

The southern bluff slope is densely vegetated with shore pine, salal, English ivy, and brush. The dunes on the beach are sparsely vegetated with European Beach grass.

2.5 Subsurface Materials

Detailed descriptions and analyses of geology and subsurface materials at the site are provided in Sections 3.0 and 3.2 below.

2.6 Site Oceanfront Conditions

The southern portion of the site is located along an oceanfront/bayfront bluff slope consisting primarily of marine terrace sands that have undergone recession due to wave, wind, and rain erosion, sloughing, and shallow landsliding. A detailed description of the fronting beach area is provided in Section 3.2, with oceanfront slope stability and erosion discussed in Section 4.0 below.

2.7 Drift Logs or Flotsam

At the time of our site visit, we observed a moderate to heavy accumulation of driftwood and flotsam in the beach area at the site. Satellite imagery indicates that the accumulation of drift logs or flotsam in the vicinity is generally consistent with slightly greater amounts of accumulation in late spring.

2.8 Streams or Drainage and Influence on Beach Elevations

The nearest major stream is the Siletz River, immediately south of the site, which outlets to Siletz Bay. The beach elevations near the site are heavily influenced by the estuary and spit and the mouth of the bay. The river has the potential to erode the base of the bluffs, potentially increasing bluff recession rates.

2.9 Headland Proximity and Influence on Beach Sediment Transport and Elevations

Headlands are not present in this local section of the Oregon Coast and the Lincoln City oceanfront. The site lies within the Lincoln littoral cell. The sands within the Lincoln littoral cell are believed to have little or no transport beyond Cascade Head, approximately 8.2 miles north of the site, and Government Point, approximately 7 miles to the south (Komar, 1997).

2.10 Shore Protection Structures

The subject property does not have an oceanfront protective structure and lies in an area of bluffs that have generally not been protected by oceanfront protective structures. The nearest oceanfront protective structures are located approximately 0.35 miles north and 0.7 miles south of the site on Siletz spit.

2.11 Beach Access Pathways

Presently there is no direct access to the beach from the subject site. Public beach access is present at Taft Park, approximately 300 feet east of the site.

2.12 Human Impacts and Influence on Site Resistance to Ocean Wave Attack

Based on our observations, direct human impacts are not contributing to the alteration of the resistance of the bluff to wave attack at the site.

3.0 Geologic Mapping, Investigation and Descriptions

3.1 Geology

The site lies in an area which has been mapped as Quaternary marine terrace deposits underlain at depth by late Eocene siltstone and sandstone of the Nestucca Formation (Schlicker et al., 1973). The Quaternary marine terrace deposits consist of semi-consolidated, fine- to medium-grained, uplifted beach sand commonly overlain by unconsolidated, fine-grained stabilized dune deposits. The uplifted marine terrace sediments are typically high-energy near-shore marine deposits capped by beach sand (Kelsey et al., 1996). The Nestucca Formation consists of thin-bedded, tuffaceous siltstone and sandstone with ash and glauconitic sandstone interbeds (Schlicker et al., 1973). Locally, the Nestucca Formation is below the beach elevation.

3.2 Description of the Fronting Beach

3.2.1 Summer and Winter Average Beach Widths

The beach at the site has a width of approximately 250 feet to more than 300 feet in this area during the winter and summer, respectively, depending upon sand transport in any given year. The beach here is dynamic and frequently changes, primarily due to the effects of the mouth of Siletz Bay, the Siletz River, and El Niño and La Niña ocean conditions. Typically, the beach is broad and dissipative in summer, becoming narrower and steeper in winter, particularly during prolonged storm cycles.

3.2.2 Beach Sediment Median Grain Size

Beach sediment is primarily fine-grained to lesser medium-grained sand.

3.2.3 Summer and Winter Beach Elevations and Average Slopes

The beach fronting the site slopes south-southwest at approximately 7 degrees in the winter and a few degrees in the summer. Based on our review of beach morphology monitoring data available for a nearby (approximately 400 feet northwest) section of Oregon's coast from 1997 to 2002, beach elevations varied by 3 to 14 feet from minimum to maximum, with minor changes at the beach-bluff junction and no substantial change of the dune (Allan and Hart, 2005). The beach elevation can change substantially associated with El Niño and La Niña events. Elevations derived from the lidar provided by NOAA for the site show the elevation above the mean sea level of the beach-bluff junction at the subject property as approximately 16 feet (NAVD 88), which agrees with data from Allan and Hart (2005).

3.2.4 Rip Currents or Embayments

Rip currents and rip current embayments have formed approximately 0.5 miles north and elsewhere within the last decade, as evidenced by our review of historical aerial imagery. The site does not appear to be prone to rip currents or their embayments.

3.2.5 Offshore Rock Outcrops and Sea Stacks

Offshore rock outcrops or sea stacks are not present near the site. Mapping by Priest and Allan (2004) shows Tertiary Intrusive Basalt outcrops approximately 0.7 miles northwest of the site.

3.2.6 Depth of Beach Sand to Bedrock

We did not observe any exposed bedrock on the beach during our previous site visits. However, we estimate sand and cobble depths along the beach at this time to be about 6 feet thick. Beach sand depths here can reach 10 feet or more in some years.

3.3 Subsurface Conditions

Previously in 2018, we completed subsurface exploration with an augered boring and test pit on Tax Lot 1300. A geologist from our office logged the boring and test pits and visually classified the soils encountered according to the Unified Soil Classification System (USCS). A detailed description of subsurface conditions is provided in Appendix B, and the approximate locations of the borings are shown on Figures 3 and 4.

Materials encountered in borings generally consisted of approximately 3 to 5 feet of loose silt fill soils, underlain by loose to medium dense silty sand and variably cemented dense sand. Conditions associated with saturation were encountered at approximately 10 feet depth in the augered boring; however, at the time this appeared to be an area of perched groundwater.

The site has been subject to past development and recent grading and fill activities. Uncontrolled fill, disturbed soil, and construction debris may be up to 10 feet thick or more on Tax Lot 1300.

3.4 Structures

Structural deformation and faulting along the Oregon Coast are dominated by the Cascadia Subduction zone (CSZ), which is a convergent plate boundary extending for approximately 680 miles from northern Vancouver Island to northern California. This convergent plate boundary is defined by the subduction of the Juan de Fuca plate beneath the North America Plate and forms an offshore north-south trench approximately 60 miles west of the Oregon coast shoreline. A resulting deformation front consisting of north-south oriented reverse faults is present along the western edge of an accretionary wedge east of the trench, and a zone of margin-oblique folding and faulting extends from the trench to the Oregon Coast (Geomatrix, 1995).

The nearest mapped potentially active faults are a group of generally northwest-striking faults collectively referred to as the Siletz Bay faults (Personius et al., 2003) that are located in the area from Government Point, approximately 6.5 miles south of Siletz Spit northward to the mouth of the Siletz River, adjacent to the site. Their sense of movement and level of activity is poorly known at present. The two most distinct faults in the group are the Fishing Rock fault and the Fogarty Creek fault. The Fishing Rock fault is mapped approximately 5.7 miles south of the site near the headland of Fishing Rock (Personius et al., 2003; Priest and Allan, 2004). This fault offsets Quaternary Marine Terrace deposits by 15 feet and is downthrown to the northeast. The Fogarty Creek fault is a downthrown-north fault with 18-foot offset and is mapped approximately 6.25 miles south of the site (Personius et al., 2003; Priest and Allan, 2004).

Other mapped potentially active faults are the Yaquina Head Fault, located approximately 17.5 miles south of the site, and the Yaquina Bay Fault, located approximately 20.5 miles south of the site. The Yaquina Head Fault is an east-trending oblique fault with left-lateral strike-slip and either contractional or extensional dip-slip offset components (Personius et al., 2003). It offsets the 80,000-year-old Newport marine terrace by approximately 5 feet, indicating a relatively low rate of slip, if still active (Schlicker et al., 1973; Personius et al., 2003). The Yaquina Bay Fault is a generally east-northeast trending oblique fault that also has left-lateral strike-slip and either contractional or extensional dip-slip offset components (Personius et al., 2003). This fault is believed to extend offshore for approximately 7 to 8 miles and may be a structurally controlling feature for the mouth of Yaquina Bay (Goldfinger et al., 1996; Geomatrix, 1995). At Yaquina Bay, a 125,000-year-old platform has been displaced approximately 223 feet up-on-the-north by the Yaquina Bay Fault. This fault has the largest component of vertical slip (as much as 2 feet per 1,000 years) of any active fault in coastal Oregon or

Washington (Geomatrix, 1995). Although the age for the last movement of the Yaquina Bay Fault is not known, the fault also offsets 80,000-year-old marine terrace sediments.

4.0 Erosion and Slope Stability

The southern part of the site is a high, steep marine terrace oceanfront bluff that has formed as the result of river and ocean wave erosion and undergoes minor continuous wind and rain impacts, minor sloughing and shallow landsliding. Older small wave cuts are present in sandy colluvium at the base of the bluff slope (Appendix A). Sparse shallow landsliding is present on the middle and lower bluff slope.

The lower bluff on the southern portion of the site is subject to riverine and ocean wave erosion. Water currents in this area and the location of the bay mouth itself have fluctuated in the past. Future changes in water currents near the bay mouth could result in increased riverine and ocean wave erosion along this section of the coastline, resulting in additional landsliding along the bluff. The site also lies in an area that has been mapped as undergoing critical erosion of marine terraces and sediments (Schlicker et al., 1973). Priest (1994) has determined the average annual erosion rate for the oceanfront bluff segments in the site area as 0.05 ± 0.05 feet per year. This erosion rate was calculated by measuring the distance between existing structures and the bluff and compared to distances measured on a 1939 or 1967 vertical aerial photograph.

The site is mapped in an area of moderate to high landslide susceptibility based on the DOGAMI methodology (Burns, Mickelson, and Madin, 2016). During our past site visits, we determined that much of Tax Lot 1800 and the southeastern and eastern areas of Tax Lot 1300 appeared to have older shallow ground movement. The steep unsupported slopes along the eastern property line of the site are susceptible to erosion, sloughing, and shallow landsliding, which can fail back 5 to 10 feet or greater at a time (Appendix A). Future erosion, sloughing, and shallow landsliding along these slopes should be anticipated.

Based on mapping completed by Priest and Allan (2004), the beach and bluff slope on the southern portion (Tax Lot 1800) of the site lies within the Active Erosion Hazard Zone. The area from the upper bluff edge to approximately 20 feet northward lies in the High-Risk Coastal Erosion Hazard Zone; the next approximately 33 feet north lies in the Moderate-Risk Coastal Erosion Hazard Zone; the next approximately 36 feet lies in the Low-Risk Coastal Hazard Zone. Coastal erosion hazard zone definitions and methodology are provided below.

The methodology provided by Priest and Allan (2004) defines four coastal erosion hazard zones for bluffs of Lincoln County, Oregon, as follows:

"*The basic techniques used here are modified from Gless and others (1998), Komar and others (1999), and Allan and Priest (2001). The zones are as follows:*

1) Active hazard zone: The zone of currently active mass movement, slope wash, and wave erosion.

2) The other three zones define high-, moderate-, and low-risk scenarios for expansion of the active hazard zone by bluff top retreat. Similar to the dune-backed shorelines, the three hazard zones depict decreasing levels of risk that they will become active in the future. These hazard zone boundaries are mapped as follows:

a. High-risk hazard zone: The boundary of the high-risk hazard zone will represent a best case for erosion. It will be assumed that erosion proceeds gradually at a mean erosion rate for 60 years, maintaining a slope at the angle of repose for talus of the bluff materials.

b. Moderate-risk hazard zone: The boundary of the moderate-risk hazard zone will be drawn at the mean distance between the high- and low-risk hazard zone boundaries.

c. Low-risk hazard zone: The low-risk hazard zone boundary represents a "worst case" for bluff erosion. The worst case is for a bluff to erode gradually at a maximum erosion rate for 100 years, maintaining its slope at the angle of repose for talus of the bluff materials. The bluff will then be assumed to suffer a maximum slope failure (slough or landslide). For bluffs composed of poorly consolidated or unconsolidated sand, another worst-case scenario will be mapped that assumes that the bluff face will reach a 2:1 slope as rain washes over it and sand creeps downward under the forces of gravity. For these sand bluffs, whichever method produces the most retreat will be adopted" (Priest and Allan, 2004).

It should be noted that mapping done for the 2004 study was intended for regional planning use, not for site-specific hazard identification.

4.1 Analyses of Erosion and Flooding Potential

4.1.1 DOGAMI Beach Monitoring Data

As discussed in Section 3.2.3 above, beach monitoring data for a nearby section of Oregon's coast shows that beach elevations varied by several feet from minimum to maximum over the monitored period of 1997 to 2002 (Allan and Hart, 2005).

4.1.2 Human Activities Affecting Shoreline Erosion

Human activity has not significantly altered wave attack resistance of the bluff at this site.

4.1.3 Mass Wasting

Weathering, landsliding, recession rates, and other erosional processes at this oceanfront site are discussed in Section 4.0 above and Section 4.2.3 below.

4.1.4 Erosion Potential From Wave Runup Beyond Mean Water Elevation

Coastal erosion rates and hazard zones (as referenced in Priest and Allan, 2004) are presented in Section 4.0 above. In the bluff-backed shoreline recession methodology applicable to the subject site, wave erosion at the bluff toe and associated parameters (rock composition, vegetative/protective cover, ballistics of debris, bluff slope angle of repose, etc.) are more critical to the erosion zone and rate estimates than calculating wave runup elevation which changes with many variables such as changing beach elevations, presence of transient dunes, etc. It is the chronic nature of the wave attack hazard that undercuts the toe of the bluff, creating bluff instability.

4.1.5 Frequency of Erosion-Inducing Processes

As discussed in Section 4.0 above, the average annual erosion rate for the site is 0.05 ± 0.05 feet per year and, as also discussed in Section 4.1.3, is currently estimated at 0.10 feet per year, resulting in 6 feet of setback over a 60-year period for erosion plus a regulatory required 5-foot setback for a total of 11 feet of setback from the upper bluff edge based on erosion. However, as discussed in Section 8.2, oceanfront bluff setbacks will need to be greater than 11 feet to be protective of structures. Ocean wave, wind and rain erosion are continuous and ongoing processes which impact bluff recession. Additionally, the site lies immediately north of the mouth of Siletz Bay and the Siletz River, which can migrate northward, eroding the lower bluff slope. Future changes in water currents near the bay mouth could result in increased riverine and ocean wave erosion along this section of coastline, resulting in landsliding along the bluff. Landsliding at the subject site would cause additional recession of the upper bluff. We anticipate that future landslides could fail back 5 to 10 feet at a time if not mitigated; however, these would be very infrequent and impossible to predict when they will occur.

4.1.6 Bluff-Backed Shoreline Erosion Potential

Discussed in Section 4.0 above, including the methodology in Priest and Allan (2004).

4.1.7 Sea Level Rise

Information from NOAA's Garibaldi and Newport/South Beach monitoring stations provides an average sea level rise of approximately 2.08 mm/year between 1967 and 2022 (NOAA Tides & Currents Sea Level Trends, http://tidesandcurrents.noaa.gov/sltrends). Global climate change can also influence rates of sea-level rise (refer to Section 7.0).

4.1.8 Estimated Annual Erosion Rate

Detailed discussion of recession and estimated erosion rates is in Section 4.0 above; Priest (1994) has determined the average annual erosion rate for the bluff at the site as 0.05 ± 0.05 feet per year.

4.2 Assessment of Potential Reactions to Erosion Episodes

4.2.1 Legal Restrictions of Shoreline Protective Structures

As noted in Section 2.0 above, the subject site does not have an oceanfront protective structure. Lots in the Taft area were generally 'developed' before January 1, 1977; however, according to the Ocean Shores Viewer (http://www.coastalatlas.net/oceanshores/, accessed April 2023), the site does not appear to be Goal 18 eligible for a beachfront protective structure.

4.2.2 Potential Reactions to Erosion Events and Future Erosion Control Measures

Site geologic hazards conclusions and development recommendations are presented in Section 8.0 below, which includes the recommended oceanfront setback for foundations along with a discussion of inherent risks to development in coastal areas with characteristics such as those at the site, as presented and analyzed in Section 4.0 above. Deep foundations, oceanfront protective structures, retaining walls, underpinning of foundations, vegetation management, relocation of structures, and bioengineering can all be potential reactions and control measures to erosion events.

4.2.3 Annual Erosion Rate for the Property

Priest (1994) has determined the average annual erosion rate for the oceanfront bluff segments in the site area as 0.10 feet per year. For further information please refer to Sections 4.0 and 4.1.8 above and Section 8.2 below.

5.0 Regional Seismic Hazards

Abundant evidence indicates that a series of geologically recent large earthquakes related to the Cascadia Subduction Zone have occurred along the coastline of the Pacific Northwest.

Evidence suggests that more than 40 great earthquakes of magnitude 8 and larger have struck western Oregon during the last 10,000 years. The calculated odds that a Cascadia earthquake will occur in the next 50 years range from 7–15 percent for a great earthquake affecting the entire Pacific Northwest to about a 37 percent chance that the southern end of the Cascadia Subduction Zone will produce a major earthquake in the next 50 years (OSSPAC, 2013; OSU News and Research Communications, 2010; Goldfinger et al., 2012). Evidence suggests the last major earthquake occurred on January 26, 1700, and may have been of magnitude 8.9 to 9.0 (Clague et al., 2000; DOGAMI, 2013).

There is now increasing recognition that great earthquakes do not necessarily result in a complete rupture along the full 1,200 km fault length of the Cascadia subduction zone. Evidence in the paleorecords indicates that partial ruptures of the plate boundary have occurred due to smaller earthquakes with moment magnitudes (Mw) < 9 (Witter et al., 2003; Kelsey et al., 2005). These partial segment ruptures appear to occur more frequently on the southern Oregon coast, as determined from paleotsunami studies. Furthermore, the records have documented that local tsunamis from Cascadia earthquakes recur in clusters (~250–400 years) followed by gaps of 700–1,300 years, with the highest tsunamis associated with earthquakes occurring at the beginning and end of a cluster (Allan et al., 2015).

These major earthquake events were accompanied by widespread subsidence of a few centimeters to 1–2 meters (Leonard et al., 2004). Tsunamis appear to have been associated with many of these earthquakes. In addition, settlement, liquefaction, and landsliding of some earth materials are believed to have been commonly associated with these seismic events.

Other earthquakes related to shallow crustal movements or earthquakes related to the Juan de Fuca plate have the potential to generate magnitude 6.0 to 7.5 earthquakes. The recurrence interval for these types of earthquakes is difficult to determine from present data, but estimates of 100 to 200 years have been given in the literature (Rogers et al., 1996).

Based on the 1999 Relative Earthquake Hazard Map of the Lincoln City area (Madin and Wang, 1999), the subject site lies in an area designated as Zone C, which represents areas having low to intermediate relative hazards associated with earthquakes. The degree of relative hazard was based on the factors of ground motion amplification, liquefaction, and slope instability, with slope instability being the most critical factor at the subject site.

The subject site is mapped in an area of very strong expected earthquake shaking during an earthquake in a 500-year period (DOGAMI Oregon HazVu website, accessed April 2023). “Very Strong” is the third-highest level of a six-level gradation from “Light” to “Violent” in this mapping system.

DOGAMI's HazVu website (https://gis.dogami.oregon.gov/maps/hazvu/) has mapped the area of the site as having a low susceptibility to liquefaction. Liquefaction is most common in saturated, loose, granular soils, sand or silty sand materials. Older sediments are also more resistant to liquefaction than recently deposited sediments (Idris and Boulanger, 2008).

6.0 Flooding Hazards

Based on the 2019 Flood Insurance Rate Map (FIRM, Panel #41041C0117E), the elevated northern portion of the site lies in an area rated as Zone X, which is defined as an area determined to be outside the 0.2% annual chance floodplain. The lower bluff slope and beach area on the southern portion of Tax Lot 1800 lies in an area rated as Zone VE (EL 24), which is defined as a coastal flood zone with velocity hazard (wave action), Base Flood Elevations determined.

Based on Oregon Department of Geology and Mineral Industries mapping (DOGAMI, 2013), the beach and lower bluff slope lie within the tsunami inundation zone resulting from an 8.7 and larger magnitude Cascadia Subduction Zone (CSZ) earthquake. The higher elevation portion of the site north of the bluff slope lies outside the mapped tsunami inundation zone resulting from a 9.1 and smaller magnitude CSZ earthquake. The 2013 DOGAMI mapping is based upon five computer-modeled scenarios for shoreline tsunami inundation caused by potential CSZ earthquake events ranging in magnitude from approximately 8.7 to 9.1. The January 1700 earthquake event (discussed in Section 5.0 above) has been rated as an approximate 8.9 magnitude in DOGAMI's methodology. More distant earthquake source zones can also generate tsunamis.

7.0 Climate Change

According to most of the recent scientific studies, the Earth's climate is changing as the result of human activities, which are altering the chemical composition of the atmosphere through the buildup of greenhouse gases, primarily carbon dioxide, methane, nitrous oxide, and chlorofluorocarbons (EPA, 1998). Although there are uncertainties about exactly how the Earth's climate will respond to enhanced concentrations of greenhouse gases, scientific observations indicate that detectable changes are underway (EPA, 1998; Church and White, 2006). Global sea-level rise, caused by melting polar ice caps and ocean thermal expansion, could lead to flooding of low-lying coastal property, loss of coastal wetlands, erosion of beaches and bluffs, and saltwater contamination of drinking water. Global climate change and the resultant sea-level rise will likely impact the subject site through accelerated coastal erosion and more frequent and severe flooding. It can also lead to increased rainfall, which can result in an increase in landslide occurrence.

8.0 Conclusions and Recommendations

The main engineering geologic concerns at the site are:

1. The bluff slope on the south part (Tax Lot 1800) of the site is undergoing continuous erosion, sloughing and shallow landsliding, which can fail back 3 to 15 feet or greater at a time. Undercutting by ocean waves and lateral erosion by the Siletz Bay river mouth can cause retreat of the toe of the slope, resulting in instability and failures along the slope. These hazards are common to oceanfront property in this area.

2. The steep slopes along the eastern property line are susceptible to erosion, sloughing, and shallow landsliding, which can fail back 5 to 10 feet or greater at a time. These slopes will continue to experience erosion, sloughing, and shallow landsliding until a more stable slope angle is achieved.

3. The site has been subject to prior grading activities with areas of thick uncontrolled fill. Based on past site observations, fill, soft/loose, disturbed, and organic-rich soils approximately 10 feet deep or more are present at the site and must be removed from footing and slab areas before construction.

4. The loose surficial soils and fill soils at the site are susceptible to erosion if not mitigated.

5. The site's southern portion lies in a FEMA VE zone and is susceptible to flooding.

6. There is an inherent regional risk of earthquakes along the Oregon Coast, which could cause harm and damage structures. Ground shaking during an earthquake can cause soils to liquefy, resulting in loss of bearing capacity and structural damage. The southern part of the site also lies within a mapped tsunami inundation hazard zone. A tsunami impacting the Lincoln City area could cause harm, loss of life, and damage to structures. These risks must be accepted by the owner, future owners, developers, and residents of the site.

The following recommendations should be adhered to during design and construction:

8.1 General Recommendations

1. HGSA will need to review a complete plan set for any proposed construction on the lot. The plans will need to incorporate the recommendations included herein. Please note that these recommendations are intended for the construction of a single-family house and a swim spa. Additional recommendations or

modifications of the recommendations included herein may be needed depending on the proposed design(s).

2. Development of this property requires mitigation for potential shallow landsliding, erosion, and steep slopes, which will likely include drainage and waterproofing of the building envelope, construction of freestanding and integrated retaining walls, specialized grading, and possibly deep foundations. Stabilization of slopes above and below the proposed structure and improvements may also be required.

3. The project's civil engineer shall prepare a formal grading plan showing an estimate of the depths and extent of all proposed excavation and fill work and temporary and permanent shoring.

4. Areas not suitable for building or development are shown on Figures 3 and 4 due to their proximity to the oceanfront bluff slope and steep slopes along the eastern portion of the site. Areas that will require the use of deep foundation systems are also shown on Figures 3 and 4 and discussed below. However, to account for the thickness of unsuitable fill on Tax Lot 1300 in the area of the proposed house and reduce the amount of required over-excavation, consideration should be given to a foundation system consisting of grade beams supported on deep foundations such as augered pile for the western portion of Tax Lot 1300.

5. For construction of driveway access to the site, uncontrolled fill materials and loose soil along the road will need to be removed and replaced with properly compacted structural fill. This structural fill will need to be retained with an engineered wall(s) and wing walls.

6. Carefully control and maintain all stormwater drainage systems at the site. Plan sets should incorporate proper drainage and erosion control, as discussed in Sections 8.4, 8.5, 8.8, 8.9, 8.10, and 8.11 below.

7. A topographic survey performed by a licensed land surveyor will be required by Lincoln City to identify the bluff edge and determine the exact location of the oceanfront bluff setback. Including the recommended non-oceanfront geologic hazard setbacks on the survey may be helpful during the design and site planning.

8. Lincoln City may also require an infiltration test to design an on-site stormwater infiltration system. However, existing groundwater, dense sandy soils and steep slopes may make it difficult to infiltrate stormwater on-site.

Provided that all recommendations herein are adhered to, no adverse effects related to the proposed development are anticipated on adjacent properties.

8.2 Development and Foundation Setbacks

Per the City of Lincoln City's requirements, we have determined an 11 feet oceanfront bluff setback based on an average annual erosion rate of 0.10 ft/yr for 60 years and have added Lincoln City's required additional 5 feet. However, as discussed below, we have determined that a more appropriate oceanfront geologic hazard setback of a minimum of 20 feet from the southern bluff edge should be used for design and construction

To help mitigate future recession of the bluff caused by erosion and landsliding, we recommend that foundations and development be set back a minimum of 20 feet north of the upper bluff edge, as shown on Figures 3 and 4. This setback would allow room on the subject property to mitigate slope issues should a (less probable) larger landslide occur in the future. An approximately 10 feet area north of this geologic hazards setback, from 20 to 30 feet from the bluff edge, will require deep foundations. New structures more than 30 feet from the southern upper bluff edge can utilize standard continuous and/or isolated spread shallow foundations.

Please note the Oregon Coast is a dynamic and energetic environment. Most of the coastline is currently eroding and will continue to erode in the future. Most structures built near ocean bluffs will eventually be undermined by erosion and landsliding. The setback recommendations presented in this report are based on past average erosion rates as determined from aerial photography, and past and current geologic conditions and processes. These setbacks are intended to protect the structure(s) from bluff recession for 60 years. Geologic conditions and the rates of geologic processes can change in the future. Setbacks greater than our recommended minimum setbacks would provide the proposed structure with greater anticipated life and lower risk from some geologic hazards.

We also recommend a *top-of-slope* geologic hazards setback from the easterly-facing slopes along the site's eastern boundary (Figures 3 and 4). Structures west of the deep foundation area on Tax Lot 1300 can utilize standard continuous and isolated spread shallow foundations; however, as discussed herein, we anticipate the greatest amount of uncontrolled fill in this area.

8.3 Site Preparation

It is anticipated that excavations at the site can be completed using conventional earth-moving equipment. Unsuitable organic-rich, soft, and fill soils should be completely removed from all building areas. Uncontrolled fill, disturbed soil, and construction debris may be up to 10 feet thick or more on Tax Lot 1300.

Due to shallow slope movement hazards from disturbed soils, grading operations should maintain generally flat and level surfaces where possible. Care should also be taken during excavation along the slopes to prevent loose materials from rolling down the hill to adjacent properties. No materials should be placed on the steep slope at the site.

Any tree stumps, including the root systems, should be removed from beneath footing, slab and pavement areas, and the resulting holes backfilled with compacted non-organic structural backfill should be placed in lifts not exceeding 8 inches and compacted to a dry density of at least 95 percent of the Modified Proctor maximum dry density (ASTM D1557).

All test pits in footing, slab, and driveway areas should be excavated to their full depth and replaced with structural fill per the recommendations provided herein. The approximate location of the test pit is shown on Figures 3 and 4.

If wet weather grading is unavoidable due to construction schedules or wet soil conditions are encountered, stabilization of the subgrade soils with aggregate may become necessary. The use of clean, well-graded 1½ inch minus crushed rock fill (containing less than 5 percent material passing the No. 200 sieve) is recommended. The thickness of the applied granular fill should be sufficient to stabilize the subgrade soils.

Tree removal at the site should be done in accordance with City of Lincoln City and other local regulations. Based on our site observations, from a geological and geotechnical perspective, primarily considering slope stability, hazardous and dead trees, as identified by your certified arborist, can be cut to the ground, leaving the stumps and rootballs intact. Many trees on the steep slopes along the eastern portion of the site are currently leaning and appear susceptible to falling and/or being undermined; as directed by your certified arborist, these also can be cut to the ground, leaving the stumps and rootballs intact. The trees on the southern bluff slope should generally be left intact, with limbing, windowing, and skirting of trees conducted under the direct supervision of a certified arborist. A landscape architect may be able to assist you in the selection of suitable trees and vegetation for replanting.

<u>Suitability of On-Site Soils for Fill</u>

On-site native sandy soils are not suitable for reuse as structural fill or as free-draining granular wall backfill and shall be hauled off-site and disposed of in a safe manner in accordance with state and local regulations. Excavating native material causes these soils to be disturbed. Disturbed native sandy soils are very susceptible to erosion and settlement. The erosion control practices recommended herein should be indicated on the plan set and implemented during site development.

As recommended herein, all structural fill and free-draining wall backfill should consist of imported granular aggregate.

8.4 Soil Bearing Capacities for Shallow Foundations

Figures 3 and 4 identify the areas of required geologic hazards setback, required deep foundations, and the allowable area for shallow foundations. To account for the thickness of unsuitable fill on Tax Lot 1300 in the area of the proposed house and reduce the amount of required over-excavation, consideration should be given to a foundation system consisting of grade beams supported on deep foundations in the western portion of the site.

8.4.1 Deep Foundations

We provide the following allowable micropile loads for 6 and 8-inch (drilled hole) diameter, 31 feet length, gravity grouted pile based on grout-to-ground bond strengths from Table 5-21 in the Federal Highway Administration National Highway Institutes Micropile Design and Construction Reference Manual. All micropile should be embedded a minimum of 31 feet. The allowable loads were calculated with conservative bond ultimate strength values of 12 psi (1,728 psf) for the sand from 10 to 20 feet depth and 23 psi (3,312 psf) for the lower 11 feet of dense sand, and a factor of safety of 2 (Sabatini et al., 2005). Bond ultimate strength for the upper 10 feet of loose sandy silt and fill was assumed to be negligible.

GRAVITY GROUTED PILE ALLOWABLE LOADS		
Pile (Drilled Hole) Diameter	6 inches	8 inches
Allowable Pile Loads (Compression) (FOS = 2)[a]	42 kips	56 kips
Allowable Pile Loads (Tension) (FOS = 2)[a]	27 kips	36 kips
[a] A representative of HGSA should observe pile installation operations and verify achieved embedment depths on-site. Please provide us with at least five (5) days notice prior to any needed site observations.		

Pile spacing can vary with the size and type of pile utilized, and HGSA should work with the structural engineer and architect during the design process. We defer to the project's structural engineer to complete the design of the deep foundation system. Prior to construction, the foundation contractor should provide a work plan for HGSA's review (also refer to Appendix C).

A representative of HGSA should observe all pile construction and installation operations to ensure that suitable materials have been encountered and address any issues that may arise during construction (Appendix C).

Any structures and all structural elements should be designed to meet current Oregon Residential Specialty Code (ORSC) and Oregon Structural Specialty Code (OSSC) seismic requirements.

8.4.2 Soil Bearing Capacities for Shallow Foundations

The following shallow foundation, slab, and retaining wall recommendations apply only to those areas not identified in Figures 3 and 4 as areas needing deep foundation support. Individual and/or continuous spread footings should bear in undisturbed, native, non-organic, stiff/dense soils or properly engineered and compacted structural fill.

ALLOWABLE SOIL BEARING CAPACITIES	
Allowable Dead Plus Live Load Bearing Capacity [a]	1,500 psf
Passive Resistance	200 psf/ft embedment depth
Lateral Sliding Coefficient	0.30

[a] Allowable bearing capacity may be increased by one-third for short-term wind or seismic loads.

We recommend that the house be constructed with an elevated floor and crawlspace or daylight basement design. For conventional light-frame construction*, our recommended minimum widths and embedment depths for continuous footings are as follows:

MINIMUM FOOTING WIDTHS & EMBEDMENT DEPTHS			
Number of Stories	One	Two	Three
Minimum Footing Width	15 inches	18 inches	23 inches
Minimum Exterior Footing Embedment Depth [a]	24 inches	24 inches	24 inches
Minimum Interior Footing Embedment Depth [b]	6 inches	6 inches	6 inches

[a] All footings shall be embedded as specified above, or extend below the frost line as per Table R301.2(1) of the 2021 ORSC, whichever provides greater embedment.

[b] Interior footings shall be embedded a minimum of 6 inches below the lowest adjacent finished grade, or as otherwise recommended by our firm. In general, interior footings placed on sloping or benched ground shall be embedded or set back from cut slopes in such a manner as to provide a minimum horizontal distance between the foundation component and face of the slope of one foot per every foot of elevation change.

*Please contact us for additional recommendations if brick veneer, hollow concrete masonry, or solid concrete or masonry wall construction is incorporated into the design of the house.

Isolated footings should meet Section R403.1.7 of the 2021 Oregon Residential Specialty Code (ORSC) requirements.

Deck footings should meet or exceed the minimum sizes set forth in Table R507.3.1 of 2021 ORSC.

8.5 Slabs-On-Ground

All areas beneath slabs for garages and driveways should be excavated a minimum of 6 inches into native, non-organic, firm soils or properly engineered and compacted structural fill. The exposed subgrade in the slab excavation should be cut smooth, without loose or disturbed soil and rock remaining in the excavation.

The slab excavation should then be backfilled with a minimum of 6 inches of ¾ inch minus, clean, free-draining, crushed rock placed in 8-inch lifts maximum, which are compacted to a minimum of 95 percent of the Modified Proctor (ASTM D1557). A representative of H.G. Schlicker & Associates should approve the condition of the base of the excavation prior to placing structural fill and/or forming and pouring concrete (Appendix C). Reinforcing of the slab is recommended, and the slab should be fully waterproofed in accordance with structural design considerations. Slab thickness and reinforcing should be determined in accordance with structural considerations. An underslab drainage system is recommended for all below-grade slabs, as per the architect's recommendations.

SLABS-ON-GROUND	
Minimum thickness of 3/4 inch minus crushed rock beneath slabs	6 inches
Compaction Requirements	95% ASTM D1557, compacted in 8-inch lifts maximum

8.6 Retaining Walls

For static conditions, freestanding retaining walls using free-draining granular backfill should be designed for a lateral active earth pressure expressed as an equivalent fluid weight (EFW) of 35 pounds per cubic foot, assuming level backfill. An EFW of 45 pounds per cubic foot should be used, assuming sloping backfill of 2H:1V.

At-rest retaining walls should be designed for a lateral at-rest pressure expressed as an equivalent fluid weight of 60 pounds per cubic foot, assuming level backfill behind the wall equal to a distance of at least half of the height of the wall. Walls need to be fully drained to prevent the build-up of hydrostatic pressures.

The EFWs herein assume static conditions and no surcharge loads from vehicles or structures. If surcharge loads will be applied to the retaining walls, forces on the walls resulting from these loads will need to be added to the pressures given above. For seismic loading, a unit pseudostatic force equal to 13.5 pcf $(H)^2$, where H is the height of the wall in feet, should be added to the static lateral earth pressure. The location of the pseudostatic force can be assumed to act at a distance of 0.6H above the base of the wall.

RETAINING WALL EARTH PRESSURE PARAMETERS	
Static Case, Active Wall (level backfill/grades)	35 pcf [a]
Static Case, Active Wall (2H:1V backfill/grades)	45 pcf [a]
Static Case, At-Rest Wall (level backfill/grades)	60 pcf [a]
Seismic Loading (level backfill/grades)	13.5 pcf $(H)^2$ [b]

[a] Earth pressure expressed as an equivalent fluid weight (EFW).
[b] Seismic loading expressed as a pseudostatic force, where H is the height of the wall in feet. The location of the pseudostatic force can be assumed to act at a distance of 0.6H above the base of the wall.

Imported free-draining granular backfill for walls should be placed in 8-inch horizontal lifts and machine compacted to 95 percent of the maximum dry density as determined by ASTM D1557. Compaction within 2 feet of the wall should be accomplished with lightweight hand-operated compaction equipment to avoid applying additional lateral pressure on the walls. Drainage of the retaining wall should consist of slotted drains placed at the base of the wall on the backfilled side and backfilled with free-draining crushed rock (less than 5% passing the 200-mesh sieve using a washed sieve method) protected by non-woven filter fabric (Mirafi® 140N or equivalent) placed between the native soil and the backfill. Filter fabric protected free-draining crushed rock should extend to within 2 feet of the ground surface behind the wall, and the filter fabric should be overlapped at the top per the manufacturer's recommendations. All walls should be fully drained to prevent the build-up of hydrostatic pressures. All retaining walls should have a minimum of 2 feet of embedment at the toe or be designed without passive resistance. The EFWs provided herein assume that free-draining material (less than 5% passing the 200-mesh sieve on a wet sieve analysis) will be used for the retaining wall backfill.

Pile-Supported Retaining Walls

Lateral loads may be resisted by passive pressures acting on embedded retaining wall footings, micropile anchors, or the use of batter pile. Batter pile and micropile anchors may be designed by the structural engineer using the grout-to-ground bond strength values presented herein.

8.7 Seismic Requirements

The structure and all structural elements should be designed to meet current Oregon Residential Specialty Code (ORSC) seismic requirements. Based on our knowledge of subsurface conditions at the site and our analysis using the guidelines recommended in the ORSC, the structure should be designed to meet the following seismic parameters:

SEISMIC DESIGN PARAMETERS	
Site Class	D
Seismic Design Category	D_2
Mapped Spectral Response Acceleration for Short Periods	S_S = 1.353 g
Site Coefficients	F_a = 1.200
Design Spectral Response Acceleration at Short Periods	S_{DS} = 1.082 g

8.8 Structural Fills

Structural fills should consist of imported, crushed granular material, free of organics and deleterious materials, and contain no particles greater than 1 inch in diameter so that nuclear methods (ASTM D2922 & ASTM D3017) can be easily used for field density and moisture testing. Structural fill should be placed and compacted in 8-inch lifts maximum and compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557. All areas to receive fill should be stripped of all soft soils, organic soils, organic debris, existing fill, and disturbed soils.

Proper test frequency and earthwork documentation usually require daily observation during stripping, rough grading, and placement of structural fill. Field density testing should generally conform to ASTM D2922 and D3017, or D1556. To minimize the number of field and laboratory tests, fill materials should be from a single source and of a consistent character. Structural fill should be approved and periodically observed by HGSA and tested by a qualified testing firm. Test results will need to be reviewed and approved by HGSA. We recommend that at least three density tests be performed for every 18 inches or every 200 cubic yards of fill placed, whichever requires more testing. Because testing is performed on an on-call basis, we recommend that the earthwork contractor schedule the testing. Relatively more testing is typically necessary on smaller projects.

STRUCTURAL FILL	
Compaction Requirements	A minimum of 95% ASTM D1557, compacted in 8-inch lifts maximum, at or near the optimum moisture content.
Benching Requirements [a]	Slopes steeper than 5H:1V that are to receive fill shall be benched. Fills shall not be placed along slopes steeper than 3H:1V, unless approved by H.G. Schlicker & Associates, Inc.
[a] Benches shall be cut into native, non-organic, firm soils. Benches shall be a minimum of 6 feet wide with side cuts no steeper than 1H:1V and no higher than 6 feet. The lowest bench shall be keyed in a minimum of 2 feet into native, non-organic, firm soils.	

8.9 Groundwater

Groundwater was encountered in our subsurface exploration and may be encountered in excavations. If groundwater is encountered, unwatering of the excavation is required and should be the contractor's responsibility. This can typically be accomplished by pumping from one or more sumps, or daylighting excavations to drain.

Permanent groundwater mitigation at the site may be necessary; consideration should be given to using a sump pump system integrated into the crawlspace.

8.10 Erosion Control

Vegetation should be removed only as necessary, and exposed areas should be replanted following construction. Disturbed ground surfaces exposed during the wet season (November 1 through April 30) should be temporarily planted with grasses or protected with erosion control blankets or hydromulch.

Temporary sediment fences should be installed downslope of any disturbed areas of the site until permanent vegetation cover can be established (Figure 5). Unless approved by HGSA, the oceanfront slope should remain undisturbed.

Exposed sloping areas steeper than 3 horizontal to 1 vertical (3H:1V) should be protected with a straw erosion control blanket (North American Green S150 or equivalent) to provide erosion protection until permanent vegetation can be established. Erosion control blankets should be installed as per the manufacturer's recommendations.

8.11 Cut and Fill Slopes

Temporary unsupported cut and fill slopes less than 8 feet in height should be sloped no steeper than 1½ horizontal to 1 vertical (1½ H:1V). If temporary slopes greater than 8 feet high are desired or water seepage is encountered in cuts, HGSA should be contacted to provide additional recommendations. Temporary cuts in excess of 5 feet high and steeper than 1½ H:1V will likely require appropriate shoring to provide for worker safety, per OSHA regulations. Temporary cuts should be protected from inclement weather by covering them with plastic sheeting to help prevent erosion.

If the cut slope recommendations provided herein cannot be achieved due to construction and/or property line constraints, temporary or permanent retention of cut slopes may be required, as determined by a representative of HGSA.

TEMPORARY AND PERMANENT CUTS	
Temporary Cuts	1½ H:1V (maximum) [a]
Permanent Cuts	2H:1V (maximum) [a]
[a] All cuts greater than 9 feet high, or cuts where water seepage is encountered, shall be approved by a representative of H.G. Schlicker & Associates, Inc.	

Permanent unsupported cut and fill slopes should be constructed no steeper than 2 horizontal to 1 vertical (2H:1V). Fill slopes steeper than 2H:1V should be mechanically reinforced using geogrids or other suitable products as approved by HGSA. Areas that slope steeper than 5H:1V and are to receive fill should be benched. Benches should be cut into native, non-organic, firm soil. The lowest bench should be keyed a minimum of 2 feet into native, firm soil, and be a minimum of 6 feet wide.

8.12 Drainage

Surface water should be diverted from building foundations and walls to approved disposal points by grading the ground surface to slope away a minimum of 2 percent for 6 feet towards a suitable gravity outlet to prevent ponding near the structures. Permanent subsurface drainage of the building perimeter is recommended to prevent extreme seasonal variation in moisture content of subgrade materials and subjection of foundations and slabs to hydrostatic pressures.

Footing drains should be installed adjacent to the perimeter footings and sloped a minimum of 2 percent to a gravity outlet. A suitable perimeter footing drain system would consist of 4-inch diameter, perforated PVC pipe (typical) embedded adjacent to the bottom of footings and backfilled with approved drain rock. The type of pipe to be utilized may depend on building agency requirements and should be verified prior to construction. HGSA also recommends lining the drainage trench excavation with a geotextile filter such as Mirafi® 140N or equivalent to increase the life of the drainage system. The perimeter drain excavation should be constructed in a manner that prevents undermining of foundation or slab components or any disturbance to supporting soils.

In addition to the perimeter foundation drain system, drainage of any crawlspace areas is required. Each crawlspace should be graded to a low point for installation of a drain that is tied into the perimeter footing drain and tightlined to an approved disposal point. All crawlspaces will need to be vented as per ORSC requirements.

All roof drains should be collected and tightlined in a separate system independent of the footing drains, or an approved backflow prevention device shall be used. All roof and footing drains should be discharged to an approved disposal point. Water should not be concentrated and discharged on slopes steeper than 3H:1V. If water will be discharged to the ground surface, we recommend that energy dissipaters, such as splash blocks or a rock apron, be utilized at all pipe outfall locations. Water collected on the site should not be concentrated and discharged to adjacent properties. Water should not be disposed of along the southern bluff slope unless piped to the toe of the slope.

8.13 Plan Review and Site Observations

We should be provided the opportunity to review all site development, foundation, drainage, and grading plans prior to construction to assure conformance with the intent of our recommendations (Appendix C). The plans, details, and specifications should clearly show that the above recommendations have been implemented into the design.

A representative of HGSA should observe foundation setbacks and site foundation excavations prior and deep foundation installation prior to placing structural fill, forming and pouring concrete (Appendix C). Please provide us with at least five (5) days' notice prior to any needed site observations. There will be additional costs for these services.

9.0 Limitations

The Oregon Coast is a dynamic environment with inherent, unavoidable risks to development. Landsliding, erosion, tsunamis, storms, earthquakes, and other natural events can cause severe impacts to structures built within this environment and can be detrimental to the health and welfare of those who choose to place themselves within this environment. The client is warned that, although this report is intended to identify the geologic hazards causing these risks, the scientific and engineering communities' knowledge and understanding of geologic hazards processes is not complete. This report pertains to the subject site only and is not applicable to adjacent sites, nor is it valid for types of development other than that to which it refers. Geologic conditions, including materials, processes, and rates, can change with time. Therefore a review of the site and/or this report may be necessary as time passes to assure its accuracy and adequacy.

The subsurface information and related information depict generalized subsurface conditions only at these specific locations and at the particular time the subsurface exploration was completed. Soil and groundwater conditions at other locations may differ from the conditions at these locations. Also, the passage of time may result in a change in the soil and groundwater conditions at the site.

Our investigation was based on engineering geological reconnaissance and a limited review of published information. The data presented in this report are believed to be representative of the site. The conclusions herein are professional opinions derived in accordance with current standards of professional practice, budget, and time constraints. No warranty is expressed or implied. Site-specific performance of this site during a seismic event has not been evaluated. If you would like us to do so, please contact us. This report may only be copied in its entirety.

10.0 Disclosure

H.G. Schlicker & Associates, Inc. and the undersigned Certified Engineering Geologist have no financial interest in the subject site, the project, or the Client's organization.

11.0 References

Allan, J. C. and Hart, R., 2005, A geographical information system (GIS) data set of beach morphodynamics derived from 1997, 1998, and 2002 LIDAR data for the central to northern Oregon coast: Technical Report to the Oregon Department of Land Conservation and Development: Oregon Department of Geology and Mineral Industries, Open-File Report O-05-09, 16 pages.

Allan, J. C., Ruggiero, P., Cohn, N., Garcia, G., O'Brien, F. E., Serafin, K., Stimely, L. L. and Roberts, J. T., 2015, Coastal Flood Hazard Study, Lincoln County, Oregon: Oregon Department of Geology and Mineral Industries, Open-File Report O-15-06, 351 p.

Burns, W. J., Mickelson, K. A., and Madin, I. P., 2016, Landslide susceptibility overview map of Oregon: Oregon Department of Geology and Mineral Industries, Open-File Report O-16-02, 48 p., 1 plate

Church, J. A., and White, N. J., 2006, A 20th-century acceleration in global sea-level rise: Geophysical Research Letters, v. 22, LO1601, 4 p.

Clague, J. J., Atwater, B. F., Wang, K., Wang, Y., and Wong, I., 2000, Penrose Conference 2000 - Great Cascadia Earthquake Tricentennial, Programs Summary, and Abstracts: Oregon Department of Geology and Mineral Industries, Special Paper 33, 156 p.

DOGAMI, 2013, Tsunami inundation maps for Lincoln City South, Lincoln County, Oregon: Oregon Department of Geology and Mineral Industries, TIM-Linc-02, maps.

EPA, 1998, Climate Change and Oregon; Environmental Protection Agency, EPA 236-98-007u, 4 p.

Geomatrix Consultants, 1995, Seismic design mapping, State of Oregon, final report: Prepared for the Oregon Department of Transportation, Project No. 2442.

Goldfinger, C., Kulm, L. D., Yeats, R. S., Appelgate, B., MacKay, M. E., and Cochrane, G. R., 1996, Active strike-slip faulting and folding of the Cascadia Subduction-Zone plate boundary and forearc in central and northern Oregon: U.S. Geological Survey Professional Paper 1560, p. 223-256.

Goldfinger, C., Nelson, C. H., Morey, A. E., Johnson, J. E., Patton, J. R., Karabanov, E., Gutiérrez-Pastor, J., Eriksson, A. T., Gràcia, E., Dunhill, G., Enkin, R. J., Dallimore, A., and Vallier, T., 2012, Turbidite event history—Methods and implications for Holocene paleoseismicity of the Cascadia subduction zone: U.S. Geological Survey Professional Paper 1661–F, 170 p.

Idris, I. M., and Boulanger, R. W., 2008, Soil Liquefaction During Earthquakes: Earthquake Engineering Research Institute, 243 p.

Kelsey, H. M., Nelson, A. R., Hemphill-Haley, E., and Witter, R. C., 2005, Tsunami history of an Oregon coastal lake reveals a 4600 yr record of great earthquakes on the Cascadia subduction zone: Geological Society of America Bulletin, v. 117, no. 7/8, p. 1009-1032.

Kelsey, H. M., Ticknor, R. L., Bockheim, J. G., and Mitchell, C. E., 1996, Quaternary upper plate deformation in coastal Oregon: Geological Society of America Bulletin, v. 108, no. 7, p. 843-860.

Komar, P. D., 1997, The Pacific Northwest coast: living with the shores of Oregon and Washington. Durham: Duke University Press.

Leonard, L. J., Hyndman, R. D., and Mazzotti, S., 2004, Coseismic subsidence in the 1700 great Cascadia earthquake: Coastal estimates versus elastic dislocation models: Geological Society of America Bulletin, May/June 2004, v. 116, no. 5/6, pp. 655–670.

Madin, I. P., and Wang, Z., 1999, Relative earthquake hazard maps for selected urban areas in western Oregon: Oregon Department of Geology and Mineral Industries, Interpretive Map Series IMS-10.

Oregon Seismic Safety Policy Advisory Commission (OSSPAC), February 2013, The Oregon Resilience Plan: Reducing Risk and Improving Recovery for the next Cascadia Earthquake and Tsunami—Report to the 77th Legislative Assembly: State of Oregon Office of Emergency Management, 341 p.

OSU News and Research Communications, May 24, 2010, Odds are 1-in-3 that a huge quake will hit Northwest in next 50 years: Oregon State University, Corvallishttp://oregonstate.edu/ua/ncs/archives/2010/may/odds-huge-quake-Northwest-next-50-years

Personius, S. F., Dart, R. L., Bradley, L-A, Haller, K. M., 2003, Map and data for Quaternary faults and folds in Oregon: U.S. Geological Survey, Open-File Report 03-095, 556 p., map.

Priest, G. R., and Allan, J. C., 2004, Evaluation of Coastal Erosion Hazard Zones Along Dune and Bluff Backed Shorelines in Lincoln County, Oregon: Cascade Head to Seal Rock, Technical Report to Lincoln County: Oregon Department of Geology and Mineral Industries, Open-File Report O-04-09, 202 pages.

Priest, G. R., 1994, Chronic geologic hazard map of the Lincoln City-Wecoma Beach Area, Coastal Lincoln County, Oregon: Oregon Department of Geology and Mineral Industries, Open-File Report O-94-14, map.

Priest, G. R., Saul, I., and Diebenow, J., 1994, Explanation of chronic geologic hazard maps and erosion rate database, coastal Lincoln County, Oregon: Salmon River to Seal Rock: Oregon Department of Geology and Mineral Industries, Open-File Report 0-94-11, 45 p.

Rogers, A. M., Walsh, T. J., Kockelman, J., and Priest, G. R., 1996, Earthquake hazards in the Pacific Northwest - an overview: U.S. Geological Survey, Professional Paper 1560, p. 1- 54.

Schlicker, H. G., Deacon, R. J., Olcott, G. W., and Beaulieu, J. D., 1973, Engineering geology of Lincoln County, Oregon: Oregon Department of Geology and Mineral Industries, Bulletin 81.

Witter, R. C., Kelsey, H. M., and Hemphill-Haley, E., 2003, Great Cascadia earthquakes and tsunamis of the past 6700 years, Coquille River estuary, southern coastal Oregon: Geological Society of America Bulletin, v. 115, p. 1289-1306.

It has been our pleasure to serve you. If you have any questions concerning this report or the site, please contact us.

Respectfully submitted,

H.G. SCHLICKER AND ASSOCIATES, INC.

CERTIFIED
OREGON
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ENGINEERING GEOLOGIST
EXPIRES: 12/31/2023

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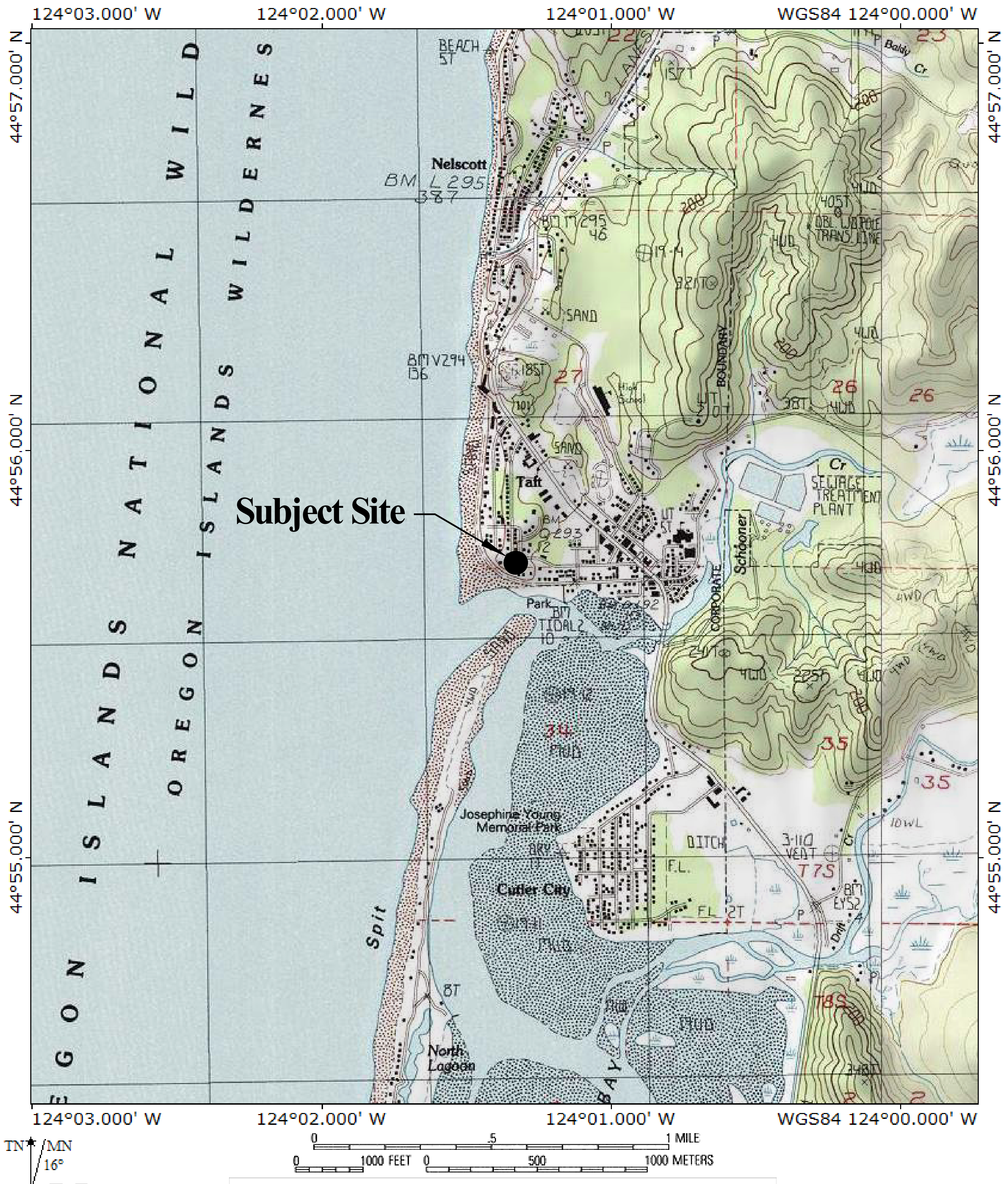


Date: 04/18/2023	**Project #Y184132C**	Prepared by: MGB
Scale: 1" = 2,000'		Approved by: AML

Location Map

Tax Lots 1300 and 1800, Map 07-11-34BA

The Pointe, Lincoln City, Oregon

H.G. Schlicker & Associates, Inc.

Figure 1

N

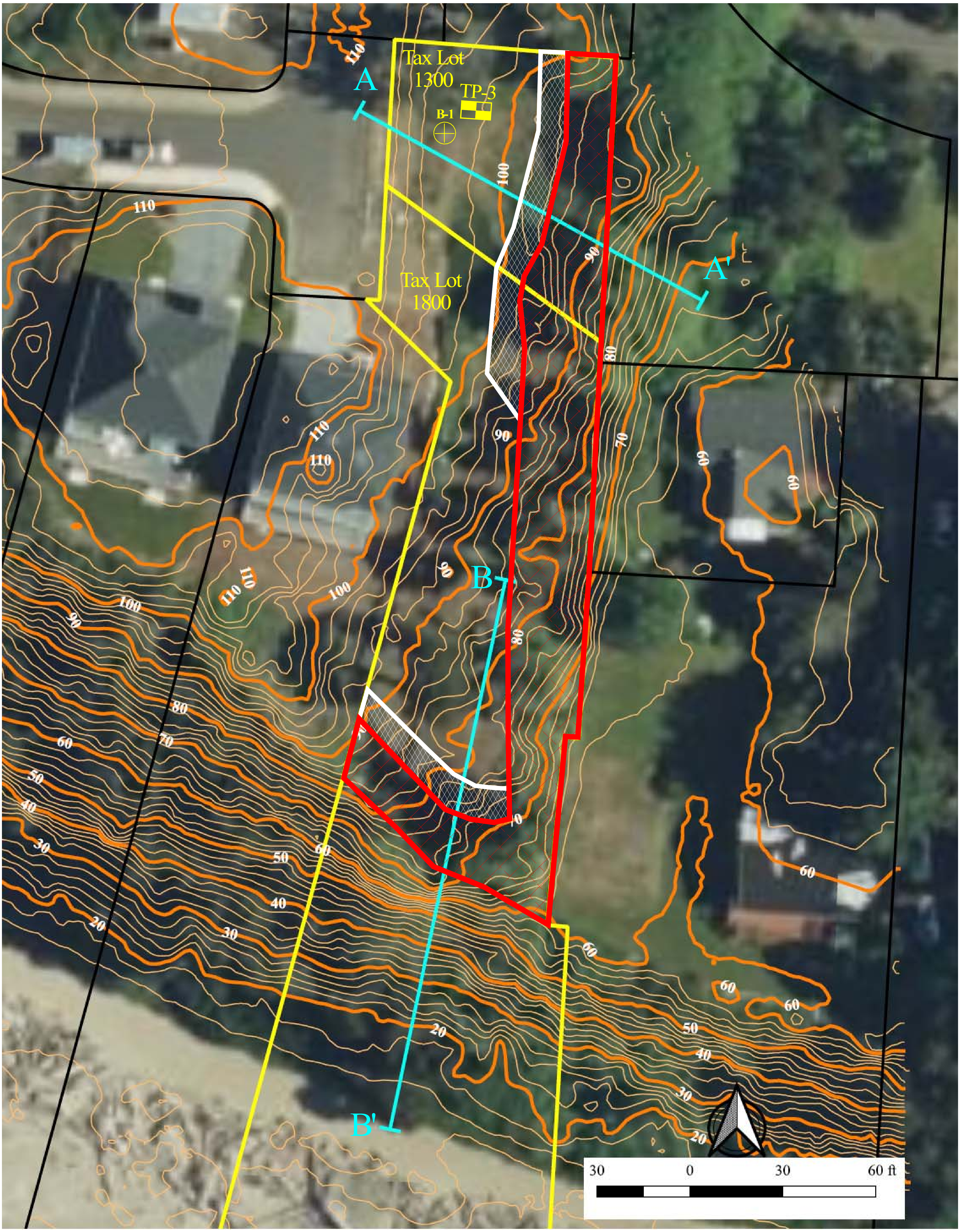
Modified from the Lincoln County assessor's plat 07-11-34BA
All locations and dimensions are approximate.

Date: 04/18/2023	**Project #Y184132C**	Prepared by: MGB
Scale: 1" = 100'		Approved by: AML

Plat Map
Tax Lots 1300 and 1800, Map 07-11-34BA
The Pointe, Lincoln City, Oregon

H.G. Schlicker & Associates, Inc.

Figure 2



A A' = Approximate trend of profile line

B-1 = Approximate location of 2018 boring

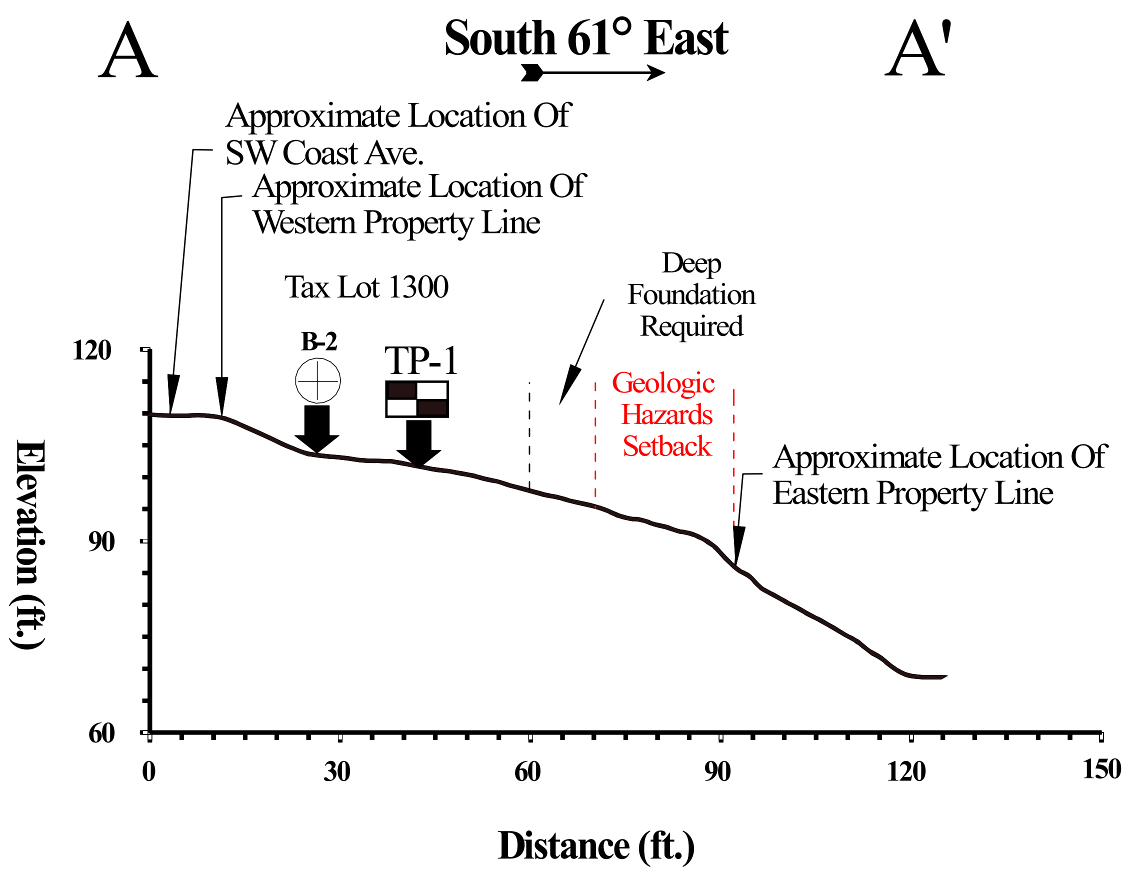
TP-1 = Approximate location of 2018 test pit

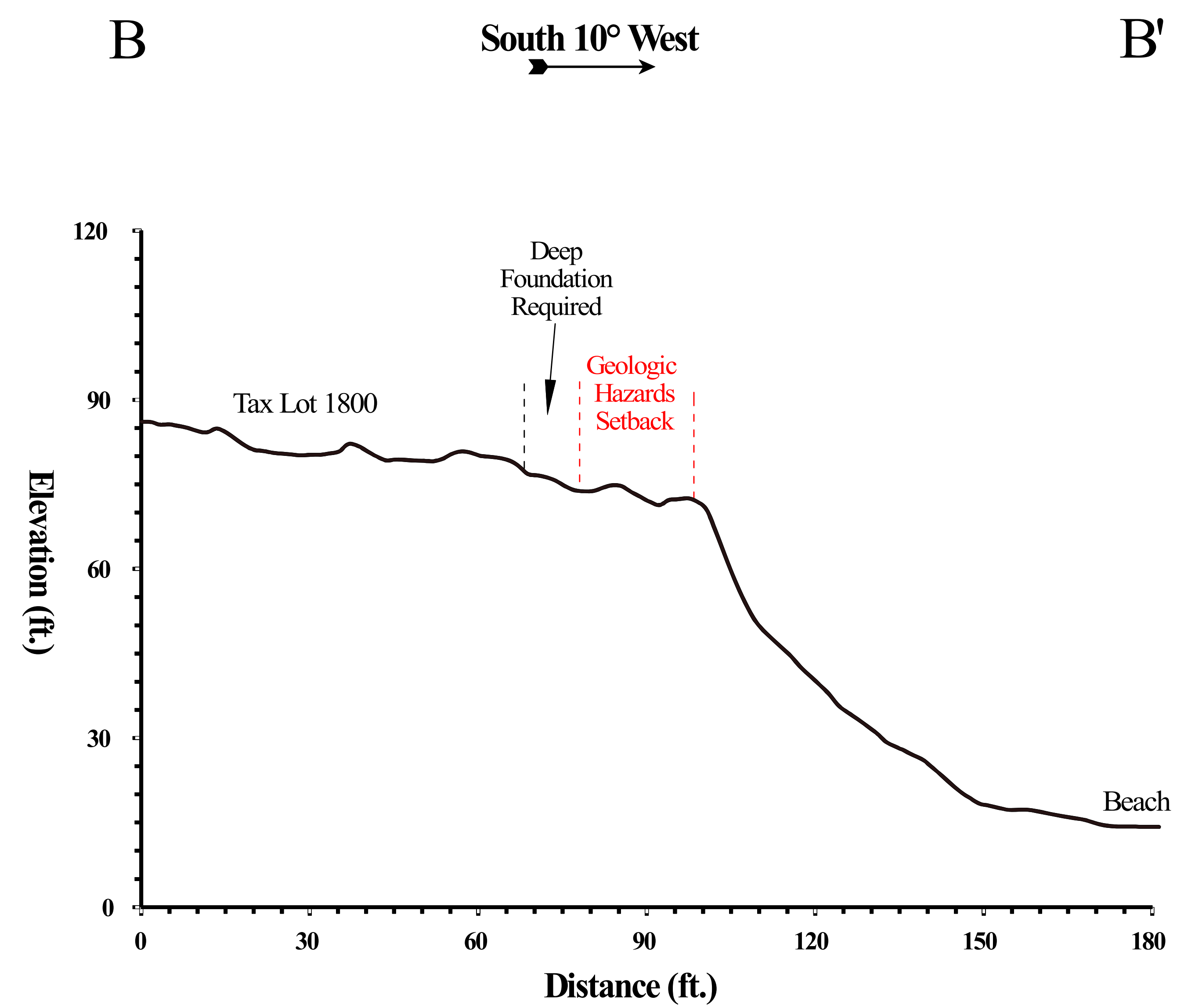
= Approximate Geologic Hazards Setback (No Foundations or Development Allowed)

= Deep Foundations Required

Imagery provided by BING.
Topographic data derived from 2016 west coast el Nino DEM lidar data from NOAA
All locations and dimensions are approximate.

Date: 04/18/2023 Scale: 1" = 30'	**Project #Y184132C**	Prepared by: MGB Approved by: AML	**Site Topographic Map** Tax Lots 1300 and 1800, Map 07-11-34BA The Pointe, Lincoln City, Oregon	H.G. Schlicker & Associates, Inc.	**Figure 3**



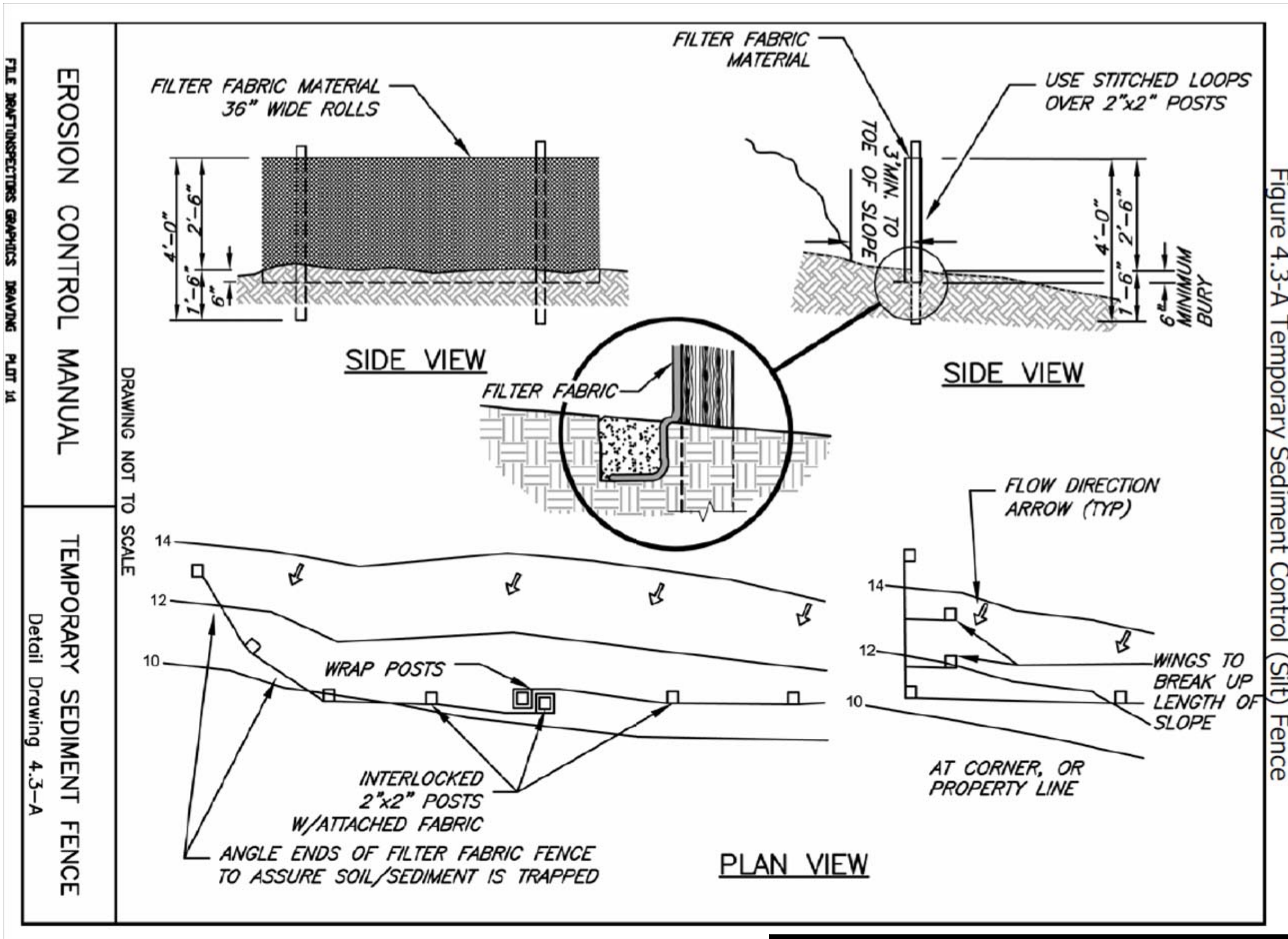


B-1 = Approximate location of 2018 boring

TP-1 = Approximate location of 2018 test pit

Slope profiles derived from 2016 west coast el Nino DEM lidar data from NOAA
All locations and dimensions are approximate.

Date: 04/18/2023 Scale: 1" = 30'	**Project #Y184132C**	Prepared by: MGB Approved by: AML
Slope Profiles, A-A' and B-B' Tax Lots 1300 and 1800, Map 07-11-34BA The Pointe, Lincoln City, Oregon		
H.G. Schlicker & Associates, Inc.		**Figure 4**



Date: 04/18/2023	**Project #Y184132C**	Prepared by: MGB
Scale: As Shown		Approved by: AML

Sediment Fence Detail

Tax Lots 1300 and 1800, Map 07-11-34BA

The Pointe, Lincoln City, Oregon

H.G. Schlicker & Associates, Inc.

Figure 5

Drawing modified from Erosion and Sediment Control Manual, City of Portland Bureau of Environmental Services, 2008.

Refer to Original Source for Design Criteria/ Specifications

Appendix A
– Site Photographs –


Photo 1 – Easterly view of the site from S.W. Coast Avenue.


Photo 2 – Northerly view of Tax Lot 1300. Note the stockpile of fill soil and debris.

Photo 3 – Northeasterly view of Tax Lot 1300 on the site's northern portion.

Photo 4 – Southerly view of Tax Lot 1800. Note the unretained cut slope.

Photo 5 – Southerly view of Tax Lot 1800.

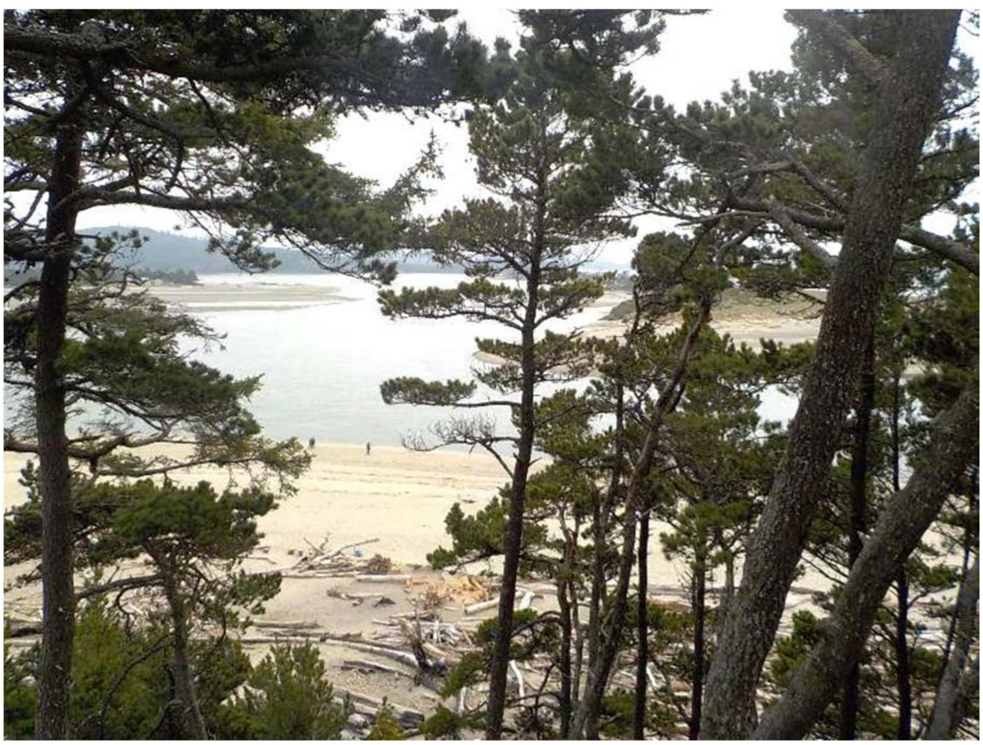


Photo 6 – View of Siletz Bay from near the bluff edge on Tax Lot 1800.

Photo 7 – Downslope view of the bluff slope and beach on the southern portion of Tax Lot 1800.

Photo 8 – Southerly view of the steep slopes along the eastern portion of Tax Lot 1800.

Photo 9 – Northerly view of the steep slopes along the eastern portion of Tax Lot 1300.

Photo 10 –Westerly view towards Tax Lot 1300 from the adjacent property.

Photo 11 – Easterly view of the large fill stockpiled on Tax Lot 1300 (3/22/2022).

Photo 12 – View of Tax Lot 1800 after brushing and clearing (3/22/2022).

Photo 13 – View of the disturbed ground surface from grading and construction activities on Tax Lot 1300 (1/31/2020).

Photo 14 – View of the disturbed ground surface from grading activities on Tax Lot 1800 (10/31/2019).

Photo 15 – View of a scarp-like feature on Tax Lot 1800 (10/15/2019).

Photo 16 – View of the drill rig set up on Tax Lot 1300 during subsurface exploration (4/12/2018).

Photo 17 – Close-up view of the native dense sands typical of that recovered in the boring (4/12/2018).

Photo 18 – View of Tax Lot 1800. Note the thick overgrown brush at that time (04/05/2018).

Photo 19 – View of a test pit excavated on Tax Lot 1300 (04/05/2018).

Photo 20 – View of the native dense sand underlying organic-rich soils and fill exposed in the test pit (04/05/2018).

Appendix B
– 2018 Subsurface Logs –

TEST PIT LOG EXPLANATION

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS), ASTM D2487			
MAJOR DIVISIONS		GROUP SYMBOL *	GROUP NAME
COARSE-GRAINED SOILS	GRAVELS	GW	Well-graded gravel
		GP	Poorly-graded gravel
		GM	Silty gravel
		GC	Clayey gravel
	SANDS	SW	Well-graded sand
		SP	Poorly-graded sand
		SM	Silty sand
		SC	Clayey sand
FINE-GRAINED SOILS	SILTS AND CLAYS Liquid Limits Less than 50	ML	Silt with low plasticity
		CL	Clay with low plasticity
		OL	Organic silt or organic clay with low plasticity
	SILTS AND CLAYS Liquid Limits 50 or more	MH	Silt with high plasticity
		CH	Clay with high plasticity
		OH	Organic silt or organic clay with high plasticity
HIGHLY ORGANIC SOILS		PT	Peat, Muck, and other highly organic soils.

* NOTE: the symbol RK (not within the USCS system) is used in our logs to denote rock materials.

2018 TEST PIT LOGS

TP-3

Depth (ft.)	USCS	Description
0 – 2.0	ML (Fill)	SILT, dark brown, wet, soft, with black organic debris and roots > 1" diameter.
2.0 – 3.0	ML (Disturbed)	CLAYEY SILT, light brown, moist, soft, with 1" roots; low plasticity. Pocket Pentrometer reading of 2.0 tons/sq. ft.
3.0 – 5.0	ML	SLIGHTLY CLAYEY SILT, light brown, moist, medium stiff; Pocket Pentrometer reading of 3.5 tons/sq. ft

BORING LOG EXPLANATION

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS), ASTM D2487			
MAJOR DIVISIONS		GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOILS	GRAVELS	GW	Well-graded gravel
		GP	Poorly-graded gravel
		GM	Silty gravel
		GC	Clayey gravel
	SANDS	SW	Well-graded sand
		SP	Poorly-graded sand
		SM	Silty sand
		SC	Clayey sand
FINE-GRAINED SOILS	SILTS AND CLAYS Liquid Limits Less than 50	ML	Silt with low plasticity
		CL	Clay with low plasticity
		OL	Organic silt or organic clay with low plasticity
	SILTS AND CLAYS Liquid Limits 50 or more	MH	Silt with high plasticity
		CH	Clay with high plasticity
		OH	Organic silt or organic clay with high plasticity
HIGHLY ORGANIC SOILS		PT	Peat, Muck, and other highly organic soils.

SAMPLE TYPE

SPT = Standard Penetration Test and Split-Barrel Sampler (ASTM D1586); 1 3/8-inch I.D.
2.5" = Modified 2.5-inch I.D. Split-Barrel Sampler.
Shelby = Thin-Walled Tube Sampler (ASTM D1587); 3-inch O.D.

Sampling Interval

= No sample attempted

= Location of retrieved sample.

= Location where sample was attempted with no recovery.

Standard Penetration Test (SPT)

Blows per 6" = Number of blows required to drive SPT sampler 6 inches using a 140 Lb. hammer dropped from a height of 30 inches (recorded in three 6" intervals).
N = Standard Penetration Resistance: Number of blows (N) required to drive SPT sampler 12 inches using a 140 Lb. hammer dropped from a height of 30 inches (ASTM D1586).
P = Indicates that SPT sampler was pushed 6 inches with only the weight of the hammer or drill stem (N = 0)

Location: South Lincoln City		**Job Name:** Picture Book		**Project #:** Y184132	
Drilling Company: Van De Hey		**Driller:** Scott		**Boring #:** B-2	
Drill Rig:	**Solid Auger**	Hollow Auger	Rotary Wash	**Sheet 1 of 3**	
Sampler Type:	**2.5" Split Barrel**	**2.8" Shelby Tube**	**SPT**	**Drilling Time**	
Drive Wt. 140 Lbs		**Fall:** 30 In.		**Start**	**Finish**
Water Level	**Depth** (ft.)	**Time**	**Date**	**Time:** 3:15 pm	**Time:** 6:00pm
				Date: 4/12/18	**Date:**
Field Personnel: M. Bordal		**Casing Depth:** (Ft.)		**Ground Elevation:** 104 (Ft.)	

Blows per 6"			N	Sample Type	Depth (Ft.)		USCS	Description
					0			B-2 locate approximately 25 feet east of S.W. Coast Ave on Lot 1300
					2.5			
6	7	7	14	SPT	5		SM	SILTY SAND, light brownish gray, moist, medium dense, m.-f. sand; 3 in layer of clayey silt. 18 inches of material recovered between 5 and 6.5 feet
					7.5			
9	9	13	22	SPT Shelby	10		SM	SILTY SAND, light brownish gray, wet, medium dense, m.-f. sand. Shelby - sand recovered was fully saturated.
					12.5			
					15			
					17.5			

Location: South Lincoln City	**Job Name:** Picture Book		**Project #:** Y184132
Drilling Company: Van De Hey	**Driller:** Scott		**Boring #:** B-2
Drill Rig:	**Solid Auger**	Hollow Auger / Rotary Wash	**Sheet 2 of 3**
Sampler Type:	**2.5" Split Barrel**	**2.8" Shelby Tube** / **SPT**	**Drilling Time**
Drive Wt. 140 Lbs	**Fall:** 30 In.		**Start** / **Finish**
Water Level	**Depth** (ft.)	**Time** / **Date**	**Time:** / **Time:**
			Date: / **Date:**
Field Personnel: M. Bordal	**Casing Depth:** (Ft.)		**Ground Elevation:** 104 (Ft.)

Blows per 6"			N	Sample Type	Depth (Ft.)		USCS	Description
9	15	18	33	SPT	20		SM	SILTY SAND, light brownish gray, wet, dense, m.-f. sand; free water in top of split spoon 16 inches of material recovered between 20 and 21.5 feet
					22.5			
					25			
					27.5			
9	16	23	39	SPT	30		SM	SILTY SAND, light brownish gray, moist, Dense, m.-f. sand. 24 inches of material recovered between 30 and 31.5 feet
					32.5			
					35			
					37.5			

Location: South Lincoln City		**Job Name:** Picture Book		**Project #:** Y184132	
Drilling Company:		**Driller:**		**Boring #:** B-2	
Drill Rig:	**Solid Auger**	Hollow Auger	Rotary Wash	**Sheet 3 of 3**	
Sampler Type:	**2.5" Split Barrel**	**2.8" Shelby Tube**	**SPT**	**Drilling Time**	
Drive Wt.	140 Lbs	**Fall:**	30 In.	**Start**	**Finish**
Water Level	**Depth** (ft.)	**Time**	**Date**	**Time:**	**Time:**
				Date: 4/12/18	**Date:**
Field Personnel:	M. Bordal	**Casing Depth:**	(Ft.)	**Ground Elevation:**	104 (Ft.)

Blows per 6"			**N**	**Sample Type**	**Depth (Ft.)**		**USCS**	**Description**
16	29	37	66	SPT	40		SM	SILTY SAND, light brownish gray, moist, very dense, m.-f. sand; 4 in layer of orange stained fine sand. 24 inches of material recovered between 40 and 41.5 feet
					42.5			
					45			
					47.5			
46	69	80	149		50		SM	SILTY SAND, light brownish gray, moist, very dense, m.-f. sand with trace of ~ 0.5" gravel. 24 inches of material recovered between 50 and 51.5 feet
								Boring terminated @ 51.5 feet upon refusal Saturated conditions encountered @ 10 feet (perched groundwater).
					52.5			
					55			
					57.5			

H.G. Schlicker & Associates, Inc.

Appendix C
– Checklist of Recommended Plan Reviews and Site Observations –

Project #Y184132C

APPENDIX C
Checklist of Recommended Plan Reviews and Site Observations
To Be Completed by a Representative of H.G. Schlicker & Associates, Inc.

Item No.	Date Done	Procedure	Timing
1*		Review site development, foundation, drainage, grading, and erosion control plans.	Prior to permitting and construction.
2*		Observe foundation excavations.	Following excavation of foundations, and prior to placing fill, forming and pouring concrete. **
3*		Observe pile installation	During construction
4*		Review Proctor (ASTM D1557) and field density test results for all fills placed at the site.	During construction.

* There will be additional charges for these services.
** Please provide us with at least 5 days' notice prior to all site observations.